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Seismic Design Criteria for Slab-Column Connections

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Two-way slabs without beams are popular floor systems because of their relatively simple formwork and the potential for shorter story heights. Earthquakes, however, have demonstrated that slab-column frames are vulnerable to brittle punching shear failures in the slab-column connection region and dropping of the slab, which are costly to repair. This paper focuses on the behavior and design of slab-column connections under combined gravity and lateral loading and reviews current design procedures, performance-based design approaches, and relevant experimental data. An equation relating the gravity shear ratio at a slab-column connection to drift capacity is presented. Finally, practical recommendations are provided for defining specific performance objectives.

Keywords: deformation capacity; effective slab width; performance-based design; punching shear.

INTRODUCTION

Two-way slabs without beams are popular floor systems because of their relatively simple formwork and the potential for shorter story heights due to their shallow profile. This structural system is common in regions of low to moderate seismic risk, where it is allowed as a lateral-force-resisting system (LFRS), as well as in regions of high seismic risk for gravity systems where moment frames or shear walls are provided as the main LFRS. Earthquakes, however, have demonstrated that slab-column frames are not suitable as a main LFRS in regions of high seismic risk because they are relatively flexible and because of the potential for brittle punching shear failures in the slab-column connection region.

In the last 40 years, a significant number of experiments have been conducted to evaluate the performance of slab-column connections under cyclic lateral loading. This information has formed the basis of current code provisions and guidelines for the design of slab-column connections under combined gravity and lateral loading. As performance-based seismic design (PBSD) becomes more common in structural engineering practice, it is important to evaluate the recommended limits for various structural systems with respect to the latest experimental data and post-earthquake observations. This paper focuses on the behavior and design of interior slab-column connections under combined gravity and lateral loading and serves to review current design procedures, PBSD approaches, and relevant experimental data. Equation (23), for drift capacity of these systems in terms of the gravity shear ratio, is derived using the collected experimental data. Finally, practical recommendations are provided for the PBSD of slab-column connections.

RESEARCH SIGNIFICANCE

The objectives of this paper, developed by a task group within ACI Committee 374, Performance-Based Seismic Design of Concrete Buildings, are: 1) to review the current state of practice and PBSD approaches for slab-column connections; 2) to summarize experimental data for slab-column connections tested under combined gravity and lateral loads; and 3) to present a practical approach for PBSD of slab-column connections. The PBSD material is presented in a format consistent with the limit states suggested in FEMA 356 (ASCE 2000) and is intended to provide guidance primarily for new construction. The criteria, however, could also be applied to existing structures that contain subpar seismic details where a moderate seismic demand is expected. As a significant benefit for design approaches outside the PBSD framework, a practical equation that relates drift capacity to gravity shear ratio is presented (Eq. (23)).

SLAB-COLUMN FRAMES AND CONNECTIONS

Slab-column frame construction can deliver several desirable architectural features, including larger open space, lower building heights for a given number of stories, and efficient construction. The FEMA 356, "Prestandard and Commentary for the Seismic Rehabilitation of Buildings" (ASCE 2000) classifies slab-column moment frames as frames that meet the following conditions:

1. Framing components shall be slabs (with or without beams in the transverse direction), columns, and their connections;
2. Frames shall be of monolithic construction that provides for moment transfer between slabs and columns; and
3. Primary reinforcement in slabs contributing to lateral load resistance shall include nonprestressed reinforcement, prestressed reinforcement, or both.

This classification includes both frames that are or are not intended to be part of the LFRS for new, existing, and rehabilitated structures.

The connections between the slab and a column can be accomplished in several ways including direct connection (whether from solid or waffle slab construction), with column drop panels, and with column or shear capitals. Shear capitals are provided to increase the shear capacity at the slab-column connection and are defined by Joint ACI-ASCE Committee 352 (1989) as a thickened portion of the slab around a column that does not meet the ACI 318 plan dimension requirements for drop panels. A column capital is defined as a flared portion of the column below the slab that is cast monolithically with the slab.

Slab-column connections in structures subjected to earthquake or wind loading must transfer forces due to both gravity and lateral loads. This combination can create large shear and unbalanced moment demands at the connection. Without proper detailing, the connection can be susceptible to two-way (punching) shear failure during response to lateral loads. The flexibility of a slab-column frame can lead to large lateral deformations, which may increase the potential

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for punching failures; therefore, in regions of high seismic risk, slab-column frames are used in conjunction with beam-column moment frames or shear walls. Compatibility of lateral deformations between the slab-column frame and the LFRS, however, must be considered to determine the demands on the connections.

The seismic performance of reinforced concrete structures with flat-slab construction has demonstrated the vulnerabilities of the system. For example, following the 1985 Mexico City earthquake, punching shear failures were noted in a 15-story building with waffle flat-plate construction (Rodriguez and Diaz 1989). This failure was partly attributed to a high flexibility combined with low-ductility capacities of the waffle slab-to-column connection. In a department store during the 1994 Northridge earthquake, discontinuous flexural reinforcement at slab-column connections led to punching failures at column drop panels (Holmes and Somers 1996). Punching failures around shear capitals were also noted in the post-tensioned floor slabs of a four-story building during the same event (Hueste and Wight 1997).

CURRENT DESIGN APPROACH

General

The shear strength of slabs in the vicinity of columns is governed by the more severe of two conditions, either beam action or two-way action. In beam action, the slab acts as a wide beam with the critical section for shear extending across the entire width of the slab. This critical section is assumed to be located at a distance d (effective slab depth) from the face of the column or shear capital. For this condition, conventional beam theory applies and will not be discussed in detail herein. For the condition of two-way action, the critical section is assumed to be located at a distance $d/2$ from the perimeter of the column or shear capital, with potential diagonal tension cracks occurring along a truncated cone or pyramid passing through the critical section (refer to Fig. 1, where d_1

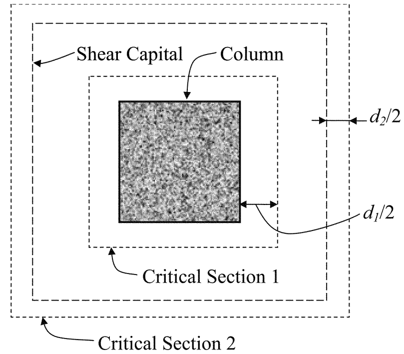

Fig. 1—Critical sections for two-way shear for interior slab-column connection with shear capital.

is the effective slab depth within the thickened shear capital region and d_2 is the effective slab depth).

Existing methods for calculating the shear strength of slab-column connections include applications of elastic plate theory, beam analogies, truss analogies, strip design methods, and others. The design method specified by ACI 318-05 (ACI Committee 318 2005) provides acceptable estimates of shear strength with reasonable computational effort. The procedure is based on the results of a significant number of experimental tests involving slab-column specimens.

The eccentric shear stress model is the basis of the general design procedure embodied in ACI 318 for determining the shear strength of slab-column connections transferring shear and moment. The model was adopted by the 1971 version of the ACI 318 and only minor modifications have been included in subsequent versions. Recently, ACI 318-05 has incorporated special provisions related to the lateral-load capacity of slab-column connections in structures located in regions of high seismic risk or structures assigned to high seismic performance or design categories.

The design approach presented in this section of the paper is based on the design procedures given in ACI 318-05 complemented by ACI 421.1R-99 (Joint ACI-ASCE Committee 421 1999) and 352.1R-89 (Joint ACI-ASCE Committee 352 1989).

ACI 318 eccentric shear stress model

Slab-column connections experience very complex behavior when subjected to lateral displacements or unbalanced gravity loads. This involves transfer of flexure, shear, and torsion in the portion of the slab around the column. Combined flexural and diagonal cracking are coupled with significant in-plane compressive forces in the slab induced by the restraint of the surrounding unyielding slab portions.

Relatively simple design equations have been derived by considering the critical section to be located at $d/2$ away from the face of the column and by assuming that shear stress on the critical perimeter varies linearly with distance from the centroidal axis. This eccentric shear stress model is based on the work by DiStasio and Van Buren (1960) and reviewed by Joint ACI-ASCE Committee 326 (1962).

For a slab-column connection transferring shear and moment, the ACI 318-05 design equations for limiting the shear stresses v_u are given by

$$v_u \leq \phi v_n \tag{1}$$

$$v_u = \frac{V_u}{b_o d} \pm \frac{\gamma_v M_u c}{J} \tag{2}$$

where v_u is the factored shear stress; ϕ is the strength reduction factor for shear; v_n is the nominal shear stress; V_u is the factored shear force acting at the centroid of the critical section; M_u is the factored unbalanced bending moment acting about the centroid of the critical section; d is the distance from the extreme compression fiber to the centroid of the longitudinal tension reinforcement; b_o is the length of the perimeter of the critical section; c is the distance from the centroidal axis of the critical section to the point where shear stress is being computed; J is a property of the critical section analogous to the polar moment of inertia; and γ_v is the fraction of the unbalanced moment considered to be transferred by eccentricity of shear, defined by

$$\gamma_v = 1 - \frac{1}{1 + \frac{2}{3}\sqrt{\frac{b_1}{b_2}}} \quad (3)$$

where b_1 and b_2 are the widths of the critical section measured in the direction of the span for which M_u is determined (Direction 1) and in the perpendicular direction (Direction 2).

For an interior column and a critical section of rectangular shape, b_o and J are determined by

$$b_o = 2(b_1 + b_2) \quad (4)$$

$$J = \frac{db_1^3}{6} + \frac{b_1 d^3}{6} + \frac{db_2 b_1^2}{2} \quad (5)$$

The first term of Eq. (2), the shear stresses due to direct shear, is assumed uniformly distributed on the critical section, and the fraction $\gamma_v M_u$ is assumed to be resisted by linear variation of shear stresses on the critical section. The portion of the moment not carried by eccentric shear is to be carried by slab flexural reinforcement placed within lines $1.5h$ on either side of the column (h is the slab thickness, including drop panel, if any). This flexural reinforcement is also used to resist slab design moments within the column strip.

The provisions of the ACI 318 specify that in absence of shear reinforcement, the nominal shear strength (in stress units) carried by the concrete v_c in nonprestressed slabs is given by

$$v_c = \min \begin{cases} 4\sqrt{f_c'}\,\text{(psi)} \\ \left(2 + \frac{4}{\beta_c}\right)\sqrt{f_c'}\,\text{(psi)} \\ \left(\frac{\alpha_s d}{b_o} + 2\right)\sqrt{f_c'}\,\text{(psi)} \end{cases} \quad (6)$$

$$\text{or } v_c = \min \begin{cases} 0.33\sqrt{f_c'}\,\text{(MPa)} \\ 0.17\left(1 + \frac{2}{\beta_c}\right)\sqrt{f_c'}\,\text{(MPa)} \\ 0.083\left(\frac{\alpha_s d}{b_o} + 2\right)\sqrt{f_c'}\,\text{(MPa)} \end{cases}$$

For prestressed slabs without shear reinforcement, Eq. (6) is replaced by

$$v_c = \min \begin{cases} 3.5\sqrt{f_c'}\,\text{(psi)} + 0.3f_{pc} + \frac{V_p}{b_o d} \\ \left(\frac{\alpha_s d}{b_o} + 1.5\right)\sqrt{f_c'}\,\text{(psi)} + 0.3f_{pc} + \frac{V_p}{b_o d} \end{cases} \quad (7)$$

$$v_c = \min \begin{cases} 0.29\sqrt{f_c'}\,\text{(MPa)} + 0.3f_{pc} + \frac{V_p}{b_o d} \\ 0.083\left(\frac{\alpha_s d}{b_o} + 1.5\right)\sqrt{f_c'}\,\text{(MPa)} + 0.3f_{pc} + \frac{V_p}{b_o d} \end{cases}$$

where α_s equals 40, 30, and 20 for interior, edge, and corner columns, respectively; b_o and d are defined previously; β_c is the ratio of long side to short side of column; f_c' is the specified concrete compressive strength (psi units); f_{pc} is the average compressive stress in two vertical slab sections in perpendicular directions, after allowance for all prestress losses; and V_p is the vertical component of all effective prestress forces crossing the critical section.

The use of Eq. (7) is restricted to cases where f_c' is less than 5000 psi (35 MPa); f_{pc} ranges between 125 and 500 psi (0.9 and 3.5 MPa) in each direction; and no portion of the column cross section is closer than four times the slab thickness to a discontinuous edge. If these conditions are not satisfied, the slab should be treated as nonprestressed and Eq. (6) applies.

When $v_u > \phi v_n$, the slab shear capacity can be increased by: (a) thickening the slab in the vicinity of the column with a column capital, shear capital, or drop panel; (b) adding shear reinforcement; (c) increasing the specified compressive strength of concrete; or (d) increasing the column size. In a flat slab with shear capitals or drop panels, stresses must be checked at all critical locations—both at the thickened portion of the slab near the face of the column and at the section outside the shear capital or drop panels (refer to Fig. 1).

Shear reinforcement, which can be in the form of bars or wires and single- or multiple-leg stirrups properly anchored, increases both the shear strength and the ductility of the connection when transferring moment and shear. Shear reinforcement consisting of structural steel shapes (shearheads) is also effective in increasing the shear strength and ductility of slab-column connections. Design procedures for shearhead reinforcement are presented in Corley and Hawkins (1968) and are not discussed in this paper. For members with shear reinforcement other than shearheads, the nominal shear strength (in stress units) is calculated using

$$v_c = v_c + v_s \leq 6\sqrt{f_c'}\,\text{(psi)} \text{ or } 0.5\sqrt{f_c'}\,\text{(MPa)} \quad (8)$$

$$v_c = 2\sqrt{f_c'}\,\text{(psi)} \text{ or } 0.17\sqrt{f_c'}\,\text{(MPa)} \quad (9)$$

$$v_s = \frac{A_v f_{yv}}{b_o s} \quad (10)$$

where v_s is the nominal shear stress provided by shear reinforcement; A_v is the area of shear reinforcement; f_{yv} is the specified yield strength of shear reinforcement; s is the spacing of shear reinforcement; and v_c, f_c', and b_o are defined previously.

When lightweight aggregate concrete is used, the value of $\sqrt{f_c'}$ in Eq. (6) through (9) is multiplied by 0.75 for all lightweight concrete or by 0.85 for sand-lightweight concrete. The extent of the shear-reinforced zone is determined to ensure that punching shear failure does not occur immediately outside this region for the design actions.

The nominal ultimate concrete shear stress along the critical section acting with shear reinforcement is taken as $2\sqrt{f_c'}\text{ (psi)}(0.17\sqrt{f_c'}\text{ [MPa]})$ because at approximately this stress, diagonal tension cracks begin to form and cracking is needed to mobilize the shear reinforcement. The shear reinforcement or shear capital must be extended for a sufficient distance until the critical section outside the reinforced region satisfies Eq. (9). In nonprestressed slabs, the maximum spacing of shear reinforcement is $0.5d$. In prestressed slabs, the spacing of shear reinforcement is allowed to reach $0.75h$ but not to exceed 24 in. (0.61 m).

For both prestressed and nonprestressed slabs, ACI 318 mandates continuity reinforcement to give the slab some residual capacity following a single punching shear failure at a single support. Thus, in nonprestressed slabs, all bottom bars within the column strip shall be continuous and at least two of the column strip bottom bars in each direction shall pass through the column core (ACI Committee 318 2005, Section 13.3.8.5). In prestressed slabs, a minimum of two tendons shall be provided in each direction through the critical shear section over columns (ACI Committee 318 2005, Section 18.12.4).

ACI 421.1R-99 refinements

ACI 318 sets out the principles of design for slab shear reinforcement but does not make specific reference to mechanically anchored shear reinforcement, also referred to as shear studs. ACI 421.1R-99 (Joint ACI-ASCE Committee 421 1999) gives recommendations for the design of shear reinforcement using shear studs in slabs. This report also includes equations for calculating shear stresses on nonrectangular critical sections.

Shear studs have proven to be effective in increasing the strength and ductility of slab-column connections. ACI 421.1R-99 suggests treating a shear stud as the equivalent of a vertical branch of a stirrup and to use higher limits on some of the design parameters used in ACI 318. In particular, ACI 421.1R-99 suggests higher allowable values for v_n, v_c, s, and f_{yv}, as follows

$$v_c = v_c + v_s \le 8\sqrt{f_c'}\text{ (psi) or } 0.66\sqrt{f_c'}\text{ (MPa)} \tag{11}$$

$$v_c = 3\sqrt{f_c'}\text{ (psi) or } 0.25\sqrt{f_c'}\text{ (MPa)} \tag{12}$$

$$s \le \begin{cases} 0.75d \text{ when } \dfrac{v_u}{\phi} \le 6\sqrt{f_c'}\text{ (psi) or } 0.5\sqrt{f_c'}\text{ (MPa)} \\ 0.5d \text{ when } \dfrac{v_u}{\phi} > 6\sqrt{f_c'}\text{ (psi) or } 0.5\sqrt{f_c'}\text{ (MPa)} \end{cases} \tag{13}$$

$$f_{yv} \le 72{,}000\text{ psi (500 MPa)} \tag{14}$$

The justification for these higher values is mainly due to the almost slip-free anchorage of the studs and that the mechanical anchorage at the top and bottom of the stud is capable of developing forces in excess of the specified yield strength at all sections of the stud stem.

ACI 352.1R-89 recommendations

ACI 352.1R-89 (Joint ACI-ASCE Committee 352 1989) includes recommendations for the determination of connection proportions and details to ensure adequate performance of monolithic, reinforced concrete slab-column connections. The recommendations address connection strength, ductility, and structural integrity for resisting gravity and lateral forces.

ACI 352.1R-89 only applies to nonprestressed slab-column connections with f_c' less than 6000 psi (42 MPa), with or without drop panels or shear capitals, and without slab shear reinforcement. The provisions are limited to connections where severe inelastic load reversals are not expected, and do not apply to slab-column connections that are part of a primary LFRS in regions of high seismic risk because slab-column frames are generally considered to be inadequate for multi-story buildings in these areas.

ACI 352.1R-89 classifies slab-column connections as one of two types: 1) Type 1—connections not expected to undergo deformations into the inelastic range; and 2) Type 2—connections requiring sustained strength under moderate deformations into the inelastic range. In structures subjected to high winds or seismic loads, a slab-column connection should be classified as Type 2 even though it is not designated as part of the primary LFRS.

To ensure a minimal level of ductility, ACI 352.1R-89 references the work by Pan and Moehle (1989) and recommends that for all Type 2 connections—without shear reinforcement—the direct factored shear V_u acting on the connection, for which inelastic moment transfer is anticipated, must satisfy

$$V_u \le 0.4V_c = 0.4v_c b_o d \tag{15}$$

where v_c is determined by either Eq. (6) or (7).

The limitation defined by Eq. (15) was based on a review of test data that revealed that the deformation capacity of interior connections without shear reinforcement is inversely related to the direct shear on the connection. Connections not complying with Eq. (15) exhibit virtually no post-yield deformation capacity under lateral loading. Pan and Moehle (1989) found that when the stress due to direct shear approaches $0.4v_c$, the connection experiences a brittle failure for story drift ratios of approximately 1.5%. No additional statements are made in ACI 352.1R-89 regarding other combinations of shear stress and story drift ratio. The report states that Eq. (15) may be waived if calculations demonstrate that the imposed displacement will not induce yield in the slab system. For example, the use of structural walls may adequately limit the imposed drifts on slab-column frames such that yield at the slab-column connection may not occur.

The approach by ACI 352.1R-89 suggests that the deformation capacity of slab-column connections may be defined as a function of the shear stress due to direct shear only. This approach has been developed further by Moehle (1996) and Megally and Ghali (2000). ACI 318-05 has incorporated this

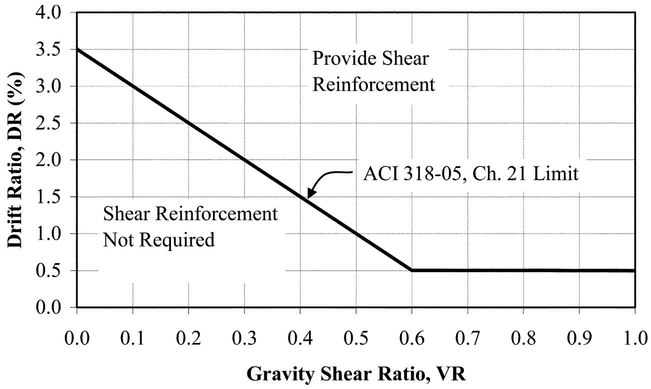

Fig. 2—ACI 318-05 relationship for determining adequacy of slab-column connections in seismic regions.

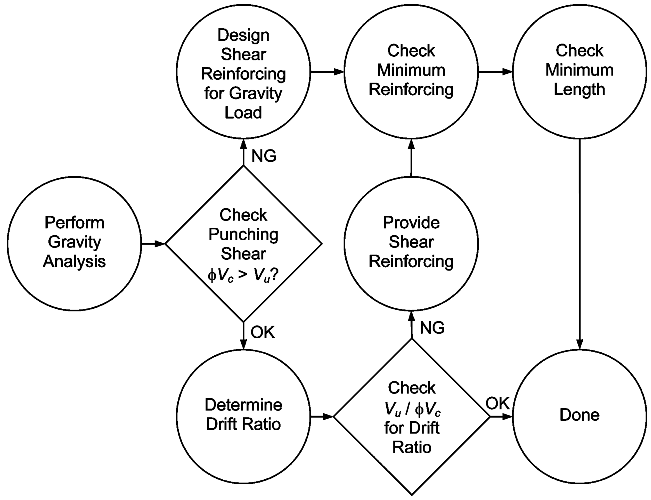

Fig. 3—Design steps when adding shear reinforcement.

concept into a general approach for addressing the deformation capacity of slab-column connections not designated as part of the LFRS.

Requirements of ACI 318-05, Section 21.11.5

Model building codes (SEI/ASCE 2005) have deformation compatibility requirements for members that are not designated as part of the LFRS. These members should be able to resist the gravity loads at lateral displacements corresponding to the design level earthquake. ACI 318-05, Section 21.11.5, has incorporated a design provision to account for the deformation compatibility of slab-column connections.

Instead of calculating the induced effects under the design displacement, ACI 318-05 describes a prescriptive approach. The connection is evaluated based on a simple relationship between the design story drift ratio (DR) and the shear stress due to factored gravity loads. The design DR (story drift divided by story height) should be taken as the largest value for the adjacent stories above and below the connection. The maximum DR (in percent) that a slab-column connection can tolerate, in the absence of shear reinforcement, is given by the following relationship and illustrated in Fig. 2.

$$DR = \begin{cases} 3.5 - 5.0VR & (\text{for } VR < 0.6) \\ 0.5 & (\text{for } VR \geq 0.6) \end{cases} \tag{16}$$

where VR is the shear ratio, defined as

$$VR = \frac{V_u}{\phi v_c b_o d} \tag{17}$$

The term v_c is calculated using Eq. (6) or (7). The factored shear force V_u on the slab critical section for two-way action is determined for the load combination $1.2D + 1.0L + 0.2S$, where D, L, and S are the dead, live, and snow loads.

If the DR exceeds the limit given by Eq. (16), shear reinforcement must be provided (or the connection can be redesigned). When adding shear reinforcement, ACI 318-05 prescribes that the term v_s, defined by Eq. (10), must exceed $3.5\sqrt{f_c'}$ (psi)$(0.29\sqrt{f_c'}$ [MPa]) and the shear reinforcement must extend at least four times the slab thickness from the face of the support. Given that this approach is relatively simple, and that the added cost of providing shear reinforcement at connections is not significant for structures designed for high seismic performance categories, use of this prescriptive approach is likely to be common. The representative design steps are shown in Fig. 3.

If shear capitals, column capitals, or drop panels are used, all potential critical sections must be investigated. ACI 318-05 does not prescribe a minimum extension of shear capitals. Wey and Durrani (1992), however, recommend a minimum length equal to two times the slab thickness from the face of the column.

ANALYTICAL MODELING

The shear stresses due to the combined factored shear and moment transferred between the slab and the column under the design displacement can be determined by creating an appropriate analytical model of the slab-column frame and directly assessing the potential for punching. Recommendations by Hwang and Moehle (2000) may be used to establish the effective stiffness of the slab and to include the impact of cracking. Hwang and Moehle (2000) recommend that the uncracked effective stiffness for a model with rigid joints, for ratios of c_2/c_1 from 1/2 to 2 and a slab aspect ratio l_2/l_1 greater than 2/3, be determined using an effective beam width represented as

$$b_{int} = 2c_1 + \frac{l_1}{3} \tag{18}$$

where b_{int} is the effective width for interior frame connections (interior connections and edge connections with bending perpendicular to the edge); c_1 and l_1 are the column dimension and slab span parallel to the direction of load being considered; and c_2 and l_2 correspond to the orthogonal direction. For exterior frame connections (corner connections and edge connections with bending parallel to the edge), half the width defined in Eq. (18) is used. Effects of cross section changes, such as slab openings, are to be considered. One way to accomplish this is to vary the width of the effective beam along the span (Hwang and Moehle 1990).

To account for cracking, a stiffness reduction factor β has been proposed by Hwang and Moehle (2000) for nonprestressed slabs and is given by

$$\beta = 4\frac{c}{l} > \frac{1}{3} \tag{19}$$

where c and l are the column dimension and slab span parallel to the load direction. Kang and Wallace (2005) recommend β = 0.5 for post-tensioned floor systems with approximate values for span-to-slab thickness ratios of 40, c_1/l_1 of 1/14, and precompression of 200 psi (1.4 MPa).

The analytical model of the slab-column frame should capture the potential for both slab yielding and connection failure due to punching as recommended in FEMA 356. Figure 4 shows an approach where yielding within the slab column strip is modeled using slab-beam elements (in this case, an elastic slab-beam with stiffness properties defined by the effective beam width model, and zero-length plastic hinges on either side of the connection). Further details of this model are described by Kang et al. (2006). Punching failures can occur if the capacity of the connection element is reached or if a limiting story drift ratio is reached for a given gravity shear ratio. Hueste and Wight (1999) suggested an approach for incorporating this behavior into a nonlinear analysis program, where, after prediction of a punching shear failure, the member behavior is modified to account for the significant reduction in stiffness and strength. Kang and Wallace (2005) suggest a direct approach by employing a limit state model.

The FEMA 356 guidelines note that the analytical model for a slab-column frame should consider all potential failures including flexure, shear, shear-moment transfer, and reinforcement development at any section. The modeling information mentioned previously gives a convenient and relatively straightforward approach to modeling the behavior of slab-column frames for nonlinear static and dynamic analysis.

PERFORMANCE-BASED DESIGN CRITERIA

A review of current practice with respect to performance-based design is needed to provide context to the material presented subsequently on performance objectives for slab-column connections. The FEMA 356 prestandard (ASCE 2000) provides analytical procedures and criteria for the performance-based evaluation of existing buildings and for designing seismic rehabilitation alternatives. This prestandard includes recommended limits for deformation capacities based on the calculated gravity shear ratio, as well as a general framework for creating performance levels and objectives.

In FEMA 356, performance levels describe limitations on the maximum damage sustained during a ground motion, while performance objectives define the target performance level to be achieved for a particular intensity of ground motion. Structural performance levels in FEMA 356 include immediate occupancy, life safety, and collapse prevention. Structures at collapse prevention are expected to remain standing, but with little margin against collapse. Structures at life safety may have sustained significant damage, but still provide an appreciable margin against collapse. Structures at immediate occupancy should have only minor damage. In FEMA 356, the Basic Safety Objective is defined as life safety-performance for the basic safety earthquake 1 (BSE-1) earthquake hazard level and collapse prevention performance for the BSE-2 earthquake hazard level. BSE-1 is the smaller event corresponding to 10% probability of exceedance in 50 years (10% in 50 years) and 2/3 of the BSE-2 (2% in 50 years) event.

For a given design event and a target performance level, FEMA 356 provides acceptance criteria when using either static or dynamic analysis based on linear and nonlinear procedures. To evaluate acceptability using linear procedures,

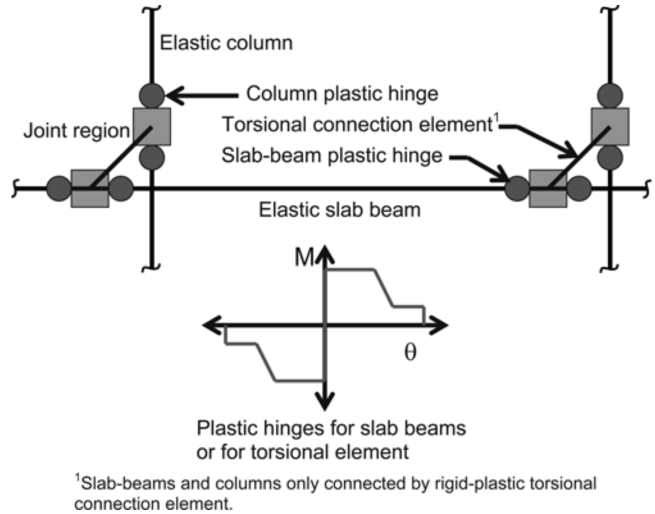

Fig. 4—Modeling of slab-column connection (adapted from Kang et al. 2006).

an action is classified as either deformation-controlled or force-controlled. Deformation-controlled actions are applicable for components that have the capacity to undergo deformations into the inelastic range without failure. Based on the demand to capacity ratio (DCR), calculated using the linear static or dynamic analysis procedures, components are classified as having low (DCR < 2), moderate (2 ≤ DCR ≤ 4), or high (DCR > 4) ductility demands.

The acceptance criteria based on linear analysis procedures are expressed in terms of m-factors. The factor m is intended to provide an indirect measure of the total deformation capacity of a structural element or component. As such, the factor m is only used to evaluate the acceptability of deformation-controlled actions

$$m\kappa Q_{CE} \geq Q_{UD} \tag{20}$$

where κ is the knowledge factor used to reduce the strength of existing components based on quality of information, Q_{CE} is the expected strength of a component or element at the deformation level considered, and Q_{UD} is the deformation-controlled design action. Equation (20) can be rearranged for direct comparison of the DCR to m to determine acceptability

$$\text{DCR} \leq m = \frac{Q_{UD}}{\kappa Q_{CE}} \tag{21}$$

The FEMA 356 limiting values for m-factors for two-way slabs and slab-column connections are provided in Table 1. The m-factors for slab-column connections range from 1 to 4 and depend on several parameters: the gravity-shear ratio, the presence of continuity reinforcement through the column cage, the development of reinforcement, and the selected performance level. The connections must also be classified as primary or secondary elements to determine the limits for life safety and collapse prevention. Secondary elements are those typically not considered to provide resistance to earthquake effects.

For nonlinear static and dynamic analysis procedures, FEMA 356 restricts inelastic response values determined from the analytical model in terms of maximum plastic rotations. Generally, plastic rotation is computed as the

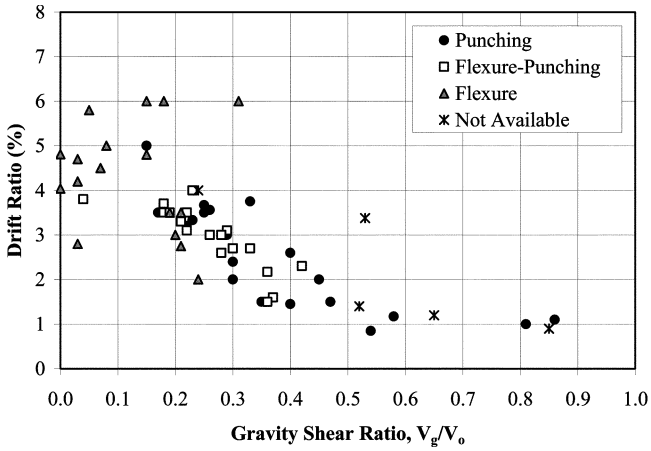

Fig. 5—Test data for interior slab-column connection specimens with no shear reinforcement.

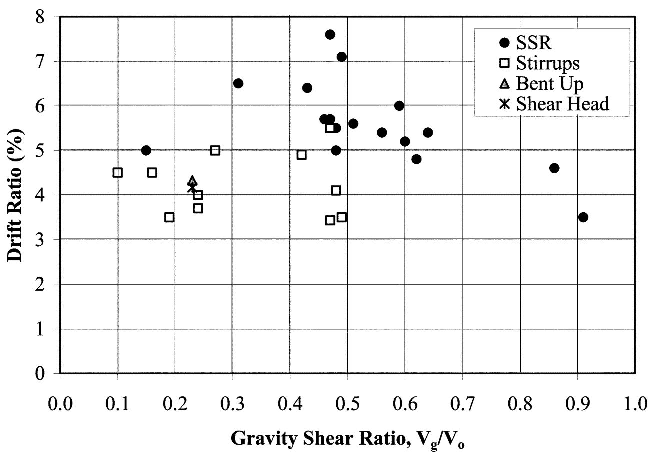

Fig. 6—Test data for interior slab-column connection specimens with shear reinforcement.

difference between the maximum rotation during analysis and the yield rotation at the member end. Therefore, it is critical for the nonlinear model to represent the maximum plastic rotation for a certain level of demand. The plastic rotation limits in FEMA 356 range from 0.0 to 0.02 radians for primary slab-column connections and from 0.0 to 0.05 radians for secondary slab-column connections. These limits are based on the gravity-shear ratio, the presence of continuity reinforcement through the column cage, the development of reinforcement, and the selected performance level (immediate occupancy, life safety, or collapse prevention).

EXPERIMENTAL DATA

Over the past 40 years, experimental studies have been conducted by researchers at a number of universities. Much of the earlier data has been summarized by Pan and Moehle (1989), Megally and Ghali (1994), and Luo and Durrani (1995). Tables 2 and 3 provide information on interior slab-column connection test specimens, with and without shear reinforcement. Limited tests have been conducted for nonductile slab-column connections where the bottom slab reinforcement is discontinuous at the interior slab-column connection (Durrani et al. 1995; Dovich and Wight 1996; Robertson and Johnson 2006) and available data is included in Table 2. The failure mode for each specimen is provided, when available, as either: punching shear *P*, flexure *F*, or a combination of flexure and punching shear (*F-P*) where a punching shear failure occurred at a higher drift level following yielding of the slab reinforcement. The gravity shear ratio and peak drift are also provided for each specimen. The peak drift is defined as the drift corresponding to the peak lateral load. Therefore, the maximum drift attained for a particular specimen may be larger than the reported peak drift.

Table 1—Acceptance criteria for linear procedures—two-way slabs and slab-column connections (adapted from FEMA 356 [ASCE 2000])

Conditions		*m*-factors by performance level*				
		IO	Component type: Primary LS	Primary CP	Secondary LS	Secondary CP
1. Slab controlled by flexure and slab-column connections†						
V_g/V_o‡	Continuity reinforcement§					
≤ 0.2	Yes	2	2	3	3	4
≥ 0.4	Yes	1	1	1	2	3
≤ 0.2	No	2	2	3	2	3
≥ 0.4	No	1	1	1	1	1
2. Slabs controlled by inadequate development or splicing along span†						
		—	—	—	3	4
3. Slabs controlled by inadequate embedment into slab-column joint†						
		2	2	3	3	4

*IO = immediate occupancy; LS = life safety; and CP = collapse prevention.

†When more than one of Conditions 1, 2, and 3 occurs for given component, use minimum appropriate numerical value from table.

‡V_g = gravity shear acting on slab critical section and V_o = direct punching shear strength as defined by ACI 318.

§Under heading "Continuity reinforcement," use "Yes" where at least one of the main bottom bars in each direction is effectively continuous through column cage. Where that slab is post-tensioned, use "Yes" where at least one of post-tensioning tendons in each direction passes through column cage. Otherwise, use "No."

The maximum drift at which an interior connection will fail can be estimated from the gravity shear ratio V_g/V_o (Pan and Moehle 1989; Luo and Durrani 1995). The gravity shear ratio represents the unfactored vertical gravity shear V_g divided by the theoretical punching shear strength without moment transfer V_o determined using

$$V_o = v_c b_o d \tag{22}$$

The term v_c is calculated using Eq. (6) or (7). A similar ratio can be computed for slabs with shear reinforcement by replacing v_c with v_n defined by Eq. (8) through (10).

Figure 5 provides a plot of peak drift as a function of V_g/V_o for interior slab-column connection specimens with no shear reinforcement. The figure shows the direct influence of the gravity shear ratio on the lateral drift capacity of slab-column connections. It may be observed that punching shear occurs for a large range of V_g/V_o values (approximately 0.1 to 0.9), while flexural failures primarily occur for V_g/V_o values of 0.3 or less.

Figure 6 provides a similar plot for interior slab-column connection specimens with shear reinforcement. The experimental data indicates that larger drift ratios are possible when shear reinforcement is used. In particular, a number of slab-column specimens with stud-shear reinforcement (SSR) attained story drift ratios well over 3% before failure.

The data from slab-column connection tests, with and without shear reinforcement, are compared in Fig. 7, along

Table 2—Test data for interior slab-column connection test specimens with no shear reinforcement

Source	Label	V_g/V_o	Peak drift, %	Mode
Dilger and Cao (1991)	CD 1	0.85	0.90	NA
	CD 2	0.65	1.20	NA
	CD 8	0.52	1.40	NA
Durrani et al. (1995)	DNY 1*	0.20	3.00	F
	DNY 2*	0.30	2.00	P
	DNY 3*	0.24	2.00	F
	DNY 4*	0.28	2.60	F-P
Elgabry and Ghali (1987)	1	0.46	NA	P
Farhey et al. (1993)	1	0.00	4.81	F
	2	0.00	4.04	F
	3	0.26	3.56	P
	4	0.30	2.40	P
Ghali et al. (1976)	SM 0.5	0.31	6.00	F
	SM 1.0	0.33	2.70	F-P
	SM 1.5	0.30	2.70	F-P
Hanson and Hanson (1968)	A12	0.29	NA	P
	A13L	0.29	NA	P
	B16	0.29	NA	P
	B7	0.04	3.80	F-P
	C17	0.24	NA	F-P
	C8	0.05	5.80	F
Hawkins et al. (1974)	S1	0.33	3.75	P
	S2	0.45	2.00	P
	S3	0.45	2.00	P
	S4	0.40	2.60	P
Hwang and Moehle (1990)	4 Int. Joints	0.24	4.00	NA
Islam and Park (1976)	1	0.25	3.67	P
	2	0.23	3.33	P
	3C	0.23	4.00	F-P
Robertson and Johnson (2006)	ND1C*	0.23	3.00 to 5.00	F-P
	ND4LL*	0.28	3.00	F-P
	ND5XL*	0.47	1.50	P
	ND6HR*	0.29	3.00	P
	NC7LR*	0.26	3.00	F-P
	ND8BU*	0.26	3.00	F-P

Source	Label	V_g/V_o	Peak drift, %	Mode
Luo and Durrani (1995)	I.I	0.08	5.00	F
	INT1	0.43	NA	P
	INT2	0.50	NA	P
Megally and Ghali (2000)	MG-2A	0.58	1.17	P
	MG-7	0.29	3.10	F-P
	MG-8	0.42	2.30	F-P
	MG-9	0.36	2.17	F-P
Morrison and Sozen (1983)	S1	0.03	4.70	F
	S2	0.03	2.80	F
	S3	0.03	4.20	F
	S4	0.07	4.50	F
	S5	0.15	4.80	F
Pan and Moehle (1989)	AP 1	0.37	1.60	F-P
	AP 2	0.36	1.50	F-P
	AP 3	0.18	3.70	F-P
	AP 4	0.19	3.50	F-P
Pan and Moehle (1992)	1	0.35	1.50	P
	2	0.35	$1.50^{EW}/0.79^{NS}$	P
	3	0.22	3.10	F-P
	4	0.22	$3.20^{EW}/1.75^{NS}$	P
Robertson and Durrani (1990)	1	0.21	2.75	F
	2C	0.22	3.50	F-P
	3SE	0.19	3.50	F
	5SO	0.21	3.50	F
	6LL	0.54	0.85	P
	7L	0.40	1.45	P
	8I	0.18	3.50	F-P
Robertson et al. (2002)	1C	0.17	3.50	P
Symonds et al. (1976)	S6	0.86	1.10	P
	S7	0.81	1.00	P
Wey and Durrani (1992)	SC 0	0.25	3.50	P
	SC 2	0.18	6.00	F
	SC 4	0.15	6.00	F
	SC 6	0.15	5.00	P
Zee and Moehle (1984)	INT	0.21	3.30	F-P

*Bottom slab reinforcement is discontinuous at interior connection.
Note: EW = east-west lateral load for biaxial test; NS = north-south lateral load for biaxial test; F = flexural failure; P = punching shear failure; and F-P = flexural and punching shear failure. NA: Not available.

with the ACI 318-05 limits for assessing the need for shear reinforcement. The line defined by ACI 318-05 is a reasonable lower-bound limit for the data corresponding to specimens without shear reinforcement. A strength reduction factor of $\phi = 1$ is used when determining V_g/V_o for the test data.

PERFORMANCE-BASED SEISMIC DESIGN RECOMMENDATIONS

Research studies and past structural performance have shown that slab-column frames provide lateral stiffness contributions to the overall LFRS and, as such, they do resist lateral loads during a seismic event even if they were designed for gravity loads only. For this reason, compatibility of deformations must be considered to calculate the demands at the slab-column connections. Likewise, the analytical model should include the strength and stiffness of the slab-column frames to ensure an accurate representation of the overall building stiffness and allow an evaluation of the magnitude of the lateral load that must be resisted by the slab-column frame members. The appropriate parameters that should be included in such a model were highlighted previously (effective slab width for equivalent beams, cracked section properties, and hysteretic behavior for nonlinear models).

Performance-based seismic design (PBSD) criteria are suggested in the following. The criteria are based on experimental data of interior slab-column connections under combined gravity and lateral load. The suggested criteria reference FEMA 356 performance levels (immediate occupancy, life safety, and collapse prevention) and seismic design requirements for slab-column connections that are adopted in ACI 318-05. As noted previously, in regions of high seismic risk, the slab-column connections of two-way

Table 3—Test data for interior slab-column connection test specimens with shear reinforcement

Source	Label	V_g/V_o	Peak drift, %	Shear reinforcement	Mode
Dilger and Brown (1995)	SJB-1	0.48	5.50	SSR	S[1]
	SJB-2	0.47	5.70	SSR	S[1]
	SJB-3	0.48	5.00	SSR	S[2]
	SJB-4	0.43	6.40	SSR	S[2]
	SJB-5	0.47	7.60	SSR	S[1]
	SJB-8	0.46	5.70	SSR	S[2]
	SJB-9	0.49	7.10	SSR	S[2]
Dilger and Cao (1991)	CD 3	0.91	3.50	SSR	NA
	CD 4	0.62	4.80	SSR	NA
	CD 6	0.64	5.40	SSR	NA
	CD 7	0.51	5.60	SSR	NA
Elgabry and Ghali (1987)	2	0.47	NA	SSR	P
	3	0.87	NA	SSR	P
	4	0.85	NA	SSR	P
	5	1.20	NA	SSR	P
Hawkins et al. (1975)	SS1	0.49	3.50	Stirrups	C[3]
	SS2	0.47	3.43	Stirrups	P
	SS3	0.48	4.10	Stirrups	F
	SS4	0.47	5.50	Stirrups	NA
	SS5	0.42	4.90	Stirrups	F
Islam and Park (1976)	4S	0.23	4.33	Bent up	P
	5S	0.23	4.17	Shear head	P
	6CS	0.24	4.00	Stirrups	P
	7CS	0.24	3.70	Stirrups	P
	8CS	0.27	5.00	Stirrups	P
Robertson et al. (2002)	2CS	0.16	4.50	Closed hoop	F
	4HS	0.15	5.00	Headed stud	F
	3SL	0.10	4.50	Single leg	F
Megally and Ghali (2000)	MG-10	0.60	5.20	SSR	NA
	MG-3	0.56	5.40	SSR	NA
	MG-4	0.86	4.60	SSR	F-P
	MG-5	0.31	6.50	SSR	F-P
	MG-6	0.59	6.00	SSR	F-P
Robertson and Durrani (1990)	4S	0.19	3.50	Closed hoop	F

Note: SSR = stud-shear reinforcement; S[1] = shear failure outside shear reinforced zone; S[2] = shear failure in shear in zones without shear reinforcement; C[3] = crushing failure at column face without apparent punching shear failure; F = flexural failure; P = punching shear failure; and F-P = flexural and punching shear failure. NA: not available.

slabs without beams must be checked for the induced effects caused by the lateral displacement expected for the design-basis earthquake. It is important to note the direct influence of the gravity shear ratio on the lateral drift capacity of slab-column connections without shear reinforcement illustrated by the test data in Fig. 5. As suggested by the FEMA 356 limits for slab-column connections, this relationship is critical to the development of appropriate PBSD criteria for slab-column connections. The ACI 318-05 seismic design limits for slab-column connections given in Eq. (16) also underscore the direct relationship between these two parameters.

Linear regression analysis on the experimental data for slab-column connections without shear reinforcement and having a gravity shear ratio V_g/V_o less than 0.6, results in a line defined by a slope of –6.95 and a zero intercept of 4.97.

Table 4—Key points for recommended PBSD criteria for interior slab-column connections

Gravity shear ratio (V_g/V_o)	Drift ratio, %, by performance level		
	IO	LS	CP
0.0	1.75	3.5	5.0
0.6	0.25	0.5	0.75
1.0	0.25	0.5	0.75

Note: IO = immediate occupancy; LS = life safety; and CP = collapse prevention.

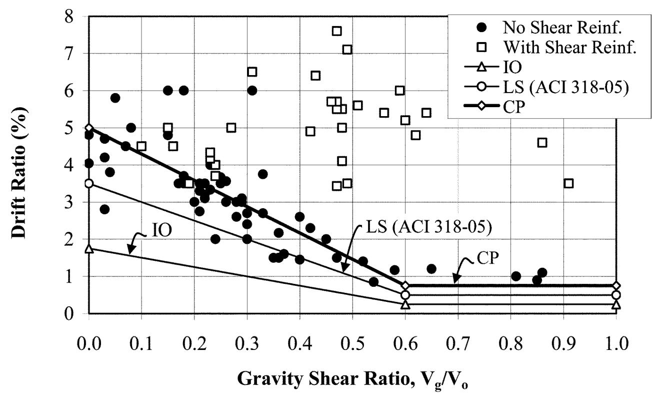

Fig. 7—Comparison of recommended performance-based seismic design limits with slab-column connection test data.

Thus, the mean for the data gives the following expression for the maximum story drift ratio (in percent)

$$DR = 5 - 7\frac{V_g}{V_o} \tag{23}$$

The PBSD criteria suggested herein use Eq. (23) as a reference for selecting the collapse prevention performance level limits. The life safety performance level was initially defined as 2/3 of the values used for collapse prevention; and for immediate occupancy, 1/3 of the values for collapse prevention was used. The drift limits determined using the aforementioned parameters were the basis for finalizing the key points of the graphed PBSD criteria. Table 4 summarizes the key points for the recommended PBSD criteria and the values are shown graphically in relationship to the test data in Fig. 7.

For the suggested PBSD criteria, the drift limits for the immediate occupancy performance level are relatively low so that the slab-column frame members remain at or near the elastic range of behavior. The suggested line for life safety corresponds to the ACI 318-05 design limits (refer to Fig. 2). The life safety performance level includes the combination of V_g/V_o = 0.4 and a drift of 1.5%, which is consistent with the recommendation in ACI 352.1R-89 that the gravity shear ratio should be kept below 0.4 to ensure some minimal ductility with the availability of approximately 1.5% drift capacity. The collapse prevention limits correspond to approximately the mean of the experimental data for specimens without shear reinforcement. For all performance levels, a constant story drift ratio capacity is assigned for gravity shear ratios in excess of 0.6.

As the approximate mean of the data for specimens without shear reinforcement (Fig. 7), the collapse prevention limits correspond to a 50% probability of failure (without considering the load and resistance factors provided in the

code). Assuming a normal distribution, the life safety limits, defined as 2/3 of collapse prevention, correspond to approximately 5% probability of failure, and the immediate occupancy limits, defined as 1/3 of collapse prevention, correspond to less than 1% probability of failure.

When the story drift limit corresponding to the acting gravity shear ratio is exceeded for the performance level considered, various options exist, including: 1) reduce the gravity shear ratio by thickening the slab, adding shear capitals, or adding drop panels; 2) reduce the story drift ratio to be within the allowable limit by stiffening the lateral system; or 3) add shear reinforcement as prescribed by ACI 318-05. For Options 1 and 2, consideration must be given to increased lateral forces resulting from the structural modification. For Option 3, the experimental data indicates that larger drift ratios are possible when shear reinforcement is used (refer to Fig. 6). The data for the shear reinforced specimens are included in Fig. 7 for comparison.

A direct comparison of the suggested PBSD criteria to the FEMA 356 acceptance criteria is not simply accomplished because FEMA limits are in terms of plastic rotations rather than drift ratios. FEMA 356 is intended for assessing existing structures and also addresses cases involving several possible deficiencies, including: 1) inadequate development or splicing along the slab span; 2) inadequate embedment into the slab-column joint; and 3) lack of continuity reinforcement through the column cage. In addition, a distinction is made between primary and secondary components. In general, the proposed PBSD limits appear to be in the range of the corresponding FEMA 356 limits. One exception is that FEMA 356 does not allow plastic rotation in primary components when the gravity shear ratio is above 0.4.

The aforementioned PBSD criteria are intended primarily for new construction. The criteria, however, could also be applied to existing structures that contain subpar seismic details where a moderate seismic demand is expected. For assessing the expected performance of a structure, the value of V_o should be computed with $\phi = 1.0$, whereas for new building design, V_o should include a strength reduction factor for shear, currently $\phi = 0.75$ in ACI 318-05.

SUMMARY AND CONCLUSIONS

This paper focuses on the behavior and design of interior slab-column connections under combined gravity and lateral loading and serves to review current design procedures, performance-based seismic design (PBSD) approaches, and relevant experimental data. Practical recommendations are provided for PBSD of slab-column connections under seismic loading conditions that can be readily implemented into design practice.

An assessment of the experimental data versus the ACI 318-05 recommendations for slab-column connections indicate that the limits for determining the necessity of slab shear reinforcement are a reasonable lower bound of the test data. Very few reports for slab-column connection specimens include plastic rotation data. FEMA 356, however, provides limits in terms of plastic rotations for nonlinear analysis procedures that are determined in part by the gravity shear ratio at the slab-column connections. The recommended PBSD criteria in this paper use two key parameters for assessing slab-column connections: the gravity shear ratio at the connection and the maximum story drift ratio. The use of story drift ratio allows a direct comparison to the experimental data and is readily available when conducting a structural analysis. A relationship between drift capacity and gravity shear ratio is provided in Eq. (23), representing an average of the collected experimental data. Three performance levels are used to match those in FEMA 356: immediate occupancy, life safety, and collapse prevention. The proposed limits correlate well with the ACI 318-05 seismic design provisions for slab-column connections and provide a practical approach for conducting PBSD for slab-column connections.

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